A design tool for piles subjected to passive displacements

Tiago G. S. Dias[1], Adam Bezuijen[1,2]

[1]*Ghent University, Ghent, Belgium*
[2]*Deltares, Delft, The Netherlands*

Abstract: The underground space of densely populated cities contains parts of buildings, utility installations, deep foundations, tunnels, and deep excavations. It is possible, and increasing more probable, that new underground constructions will be built within close proximity of existing pile foundations. This paper presents a framework for pile analysis to predict the consequences of new underground constructions on piles, specifically through the induced ground displacements. Examples for linear profiles of settlement clarify the basic mechanisms of the interaction. The settlement profiles induced by a tunnel excavation, calculated through an analytical solution, are also analysed. The results agree with the literature on pile tunnel interaction and offer a rational framework to understand the different pile responses around a tunnel excavation.

1 INTRODUCTION

The underground space of densely populated cities contains parts of buildings, utility installations, deep foundations, tunnels, and deep excavations. It is possible, and increasing more probable, that new underground constructions will be built within close proximity of existing pile foundations. Therefore, there must be a method to assess the consequences of this interaction so that it is possible to ensure the safety of the pile-supported structures. The literature exploring the case of pile tunnel interaction is quite extensive. However, the degree to which the tests and historical cases are detailed is extremely uneven. Even so, a recent study gathered empirical data to propose a few design recommendations [1], but a large degree of uncertainty still exists to estimate the pile settlements and axial forces induced by a tunnel excavation.

A closer look into the literature shows that most studies relate this interaction to the fact that the construction of a tunnel results in ground movements, and that these ground movements can influence how a pile transfers its load to the ground. Another point is that these passive displacements can increase the mobilization of the shaft friction, and if the ultimate shaft capacity is reached, significant settlements occur to remobilize the reaction of the pile toe [2]. These factors suggest that to estimate the consequences of pile tunnel interaction, the methods used for the pile analysis should be able to: a) consider the effects of passive ground settlements, and b) the possibility of full shaft mobilization. Past studies, focused on a simple version of the first requirement (a), have been able to reproduce the trends of pile/surface settlement ratios and increments of axial stress from experiments [3].

Just recently, a framework has been proposed, based on a modified version of the load transfer method, that fulfils these requirements and can be implemented in a spreadsheet with subroutines programmed in Visual Basic for Applications (VBA) [4]. This paper explores how this method can be used to understand how piles respond when subjected to passive displacements, shedding light into the mechanisms of pile tunnel interaction, specifically, and piles in interaction with new underground constructions, in general.

2 MODIFIED LOAD TRANSFER METHOD

The load-transfer method, first proposed by [5], calculates the load and settlement profiles along the pile through mobilization functions for the pile toe and at several points along the shaft. By imposing a toe displacement, vertical equilibrium of the pile segments can be iteratively calculated upwards until the pile head. The capacities of both the toe and the shaft have to be described as functions of the local pile settlement and can be bound by the pile capacity [6]. Heterogeneous ground profiles can be directly modelled by assigning different functions along the pile. These mobilization functions have, for the most part, only been calibrated for pile loading. However, there are important mechanisms taking place through the unloading stage. Irreversible deformations and residual loads are important examples related to the plasticity of the pile-soil interface and the rebound of the pile toe. Moreover, by ignoring the unloading path the models predispose the range of possible solutions for equilibrium. Therefore, two modifications are added to the general load transfer method: 1) Include a distinct unloading path in the load transfer functions, and 2) Change the variable of pile settlement for a relative pile-soil settlement, enabling the framework to consider the effects of passive displacements. The first point was adapted from the mathematical model of [7], which has been successfully applied to the analysis of bored and driven piles [8,9]. The second point has been proposed for the analysis of piles in interaction with deep excavations [10].

The variable of relative pile-soil settlement ($\Delta\delta$) is defined as the difference between the pile settlement (δp) and the soil settlement (δs) at any point along the pile, and all settlements are assumed positive downwards. So a negative $\Delta\delta$ means that the soil settles more than the pile in that point, developing a downward shear stress at the interface, also known as negative friction. At the pile toe, $\Delta\delta < 0$ indicates that there are no reaction forces from the toe, as the soil is not in contact with it. On the other hand, a positive $\Delta\delta$ is associated with upward shear, also called positive shaft friction, and an upward toe reaction. The displacements are always measured from the reference position of each point, calculated considering the pile head at the ground surface and uniform segments along the unstrained pile body. A tri-linear mobilization model is assumed for the shaft friction (Fig. 1a). The interface shear stress can be mobilized both upwards and downwards, and it was assumed that in both directions the same absolute value is achieved at full mobilization (τ_{max}). Once full mobilization is reached the model is perfectly plastic, in the sense that the displacements can continue to develop without changes in the mobilized shear stress. The model defines a transition level of mobilization (τ_{ep}) from the elastic ($S1$) to the elastoplastic ($S2$) slopes, which are defined directly through the ratios of shear mobilization to relative displacement ($\tau/\Delta\delta$). If unloading occurs after the transition level, it develops through a distinct unloading slope ($S3$), until the transition level in the opposite direction.

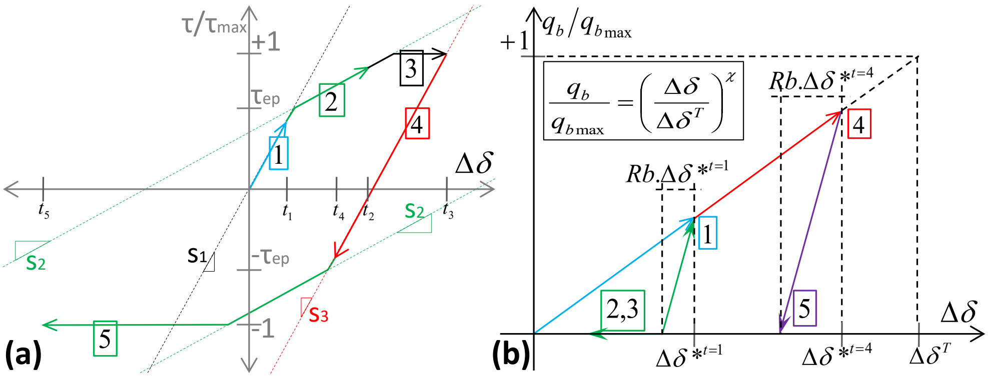

Figure 1: Mobilization models for the pile shaft (a) and toe (b).

An exponential model was proposed for the pile toe, where mobilization only occurs for positive relative displacements. For the loading branch an exponential normalized function is defined, starting at the origin ($\Delta\delta$=0, q_b=0) and reaching full mobilization ($q_{b\,max}$) at a certain relative displacement, defined as $\Delta\delta^T$. The unloading branch has to be defined in a way that does not violate the restrictions of the domain, that is to say that it shouldn't calculate toe mobilization for $\Delta\delta < 0$. Considering this limitation and the large range of displacements for toe mobilization, a rebound factor (Rb) is used to define the range of unloading. Based on these functions, any new state of equilibrium can be determined through a root search process for the relative displacement at the pile toe that satisfies equilibrium and the load boundary conditions.

3 PILE EQUILIBRIUM UNDER PASSIVE DISPLACEMENTS

In the framework of the modified load-transfer method, passive soil displacements act with the pile settlements to define the variable of relative displacements. Their balance sets the mobilization of the shaft and toe forces for equilibrium. At a certain depth, if the soil settlements are higher than the pile settlement, negative friction develops, increasing the axial force on the pile. If the soil settlements are smaller than the pile settlement, positive friction develops, reducing the axial force on the pile. The pile response due to passive displacements (PD) will always depend on the initial mobilization of the pile capacity and the associated settlements. For example, consider a 20 m long, 1 m in diameter, weightless pile supported only by friction, and a maximum shaft capacity of 1 MN obtained with a constant shear resistance along depth, and a perfectly plastic mobilization model ($S1 = S3 = 0.1$; S2 = 0; $\tau_{ep} = 1$). With a pile modulus of 10 GPa, the settlement at the pile head is 5 mm for a load of 500 kN (*WL*/*UBC* = 50%). From this loading state, a linear profile of passive displacements, with 10 mm settlement at the pile head to 0 at the pile toe, can be imposed to the pile. The results can be seen in Fig. 2.

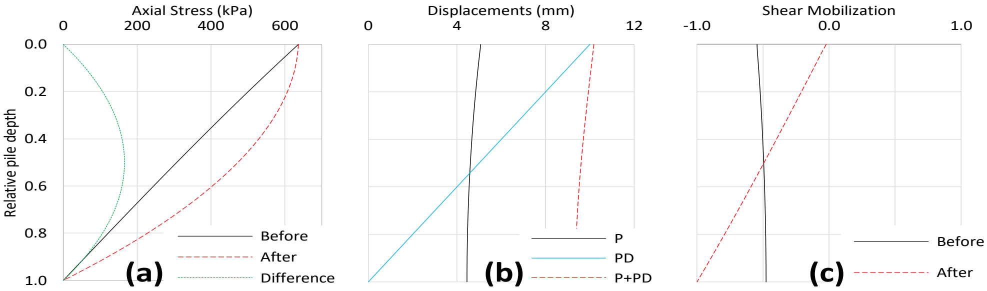

Figure 2: Example of friction pile in equilibrium under passive displacements: profiles of axial stress (a), displacements (b) and shear mobilization (c).

The profile of axial stresses shows how the effects of the passive displacements can be calculated without violating the boundary conditions of the problem (fixed head load) or the vertical equilibrium ($\sigma^n = 0$). The increment of axial stress forms a sort of parabola with the vertex around half of the pile depth. This can be understood through the profiles of settlements and shear mobilization. Before the passive displacements (PD), the pile settlements were almost uniform with depth. In relation to null ground displacements, this caused an almost uniform shear mobilization with depth. The imposition of the PD causes an additional 5 mm of settlement to the pile head. In relation to the linear profile of the PD, the pile settles the same as the ground at the surface, but the difference increases with depth. The new profile of shear mobilization is in direct relation to that difference, setting zero mobilization at the surface and practically full mobilization at the pile toe. When compared to the original profile of shear mobilization, this represents unloading in the top part of the pile and loading in the bottom part. This causes the axial stresses to increase until half of the pile depth, and decrease from there on, leading to a parabola of axial stress increments.

This example demonstrates how a simple case of a pile under passive displacements requires the simultaneous consideration of several variables. It also shows that the mobilization models have to be able to account for both loading and unloading to find the new state of equilibrium. The proposed framework can bring all these elements into the analysis and compute the consequences of any profile of passive displacements. It might also be the case that the pile head is somehow constrained by the superstructure it supports. The imminence of movement in one pile might cause a redistribution of loads through the other bearing elements of the building, such as other piles or the pile cap acting as a raft, so that global equilibrium is resolved with a different load boundary condition. Just recently, an experimental method has been proposed to link the pile settlements with the building rigidity, calculating new loading conditions for each increment of pile settlements [11,12]. The method used here can also consider the effects of passive displacements under these restrictions. Consider the previous example, but only 50% of the shaft capacity together with a toe capacity of 500 kN. The maximum toe mobilization is reached at 50 mm trough a linear toe mobilization. For these conditions, the settlement at the pile head is 8.4 mm for a load of 50% the UBC, when about 15% of the pile resistance comes from the toe.

This new set-up can be calculated assuming that the superstructure finds a new state of equilibrium with an incremental settlement at the pile head of only 2 mm. In this case, the imposed boundary conditions are the head displacement (10.4 mm) and the linear profile of passive displacements (10 mm at the surface, 0 mm at the toe level). The load at the pile head reduces from 500 kN to about 362 kN, and most of the pile shaft is de-mobilized. The toe reaction increases by 25% due to the additional settlement (Fig. 3).

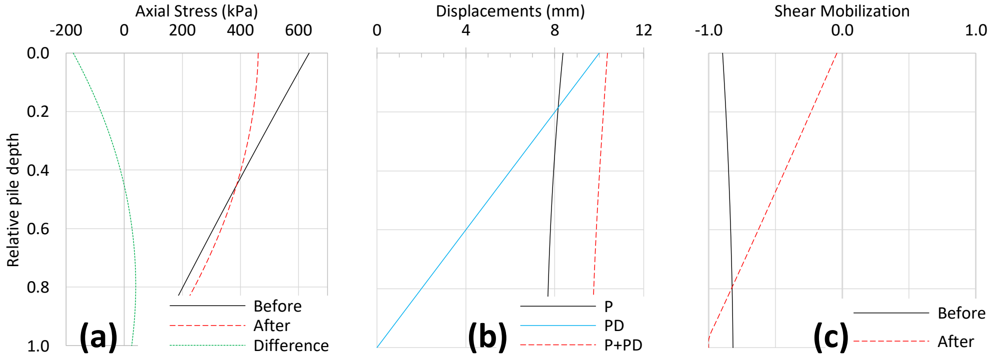

Figure 3: Example of friction pile in equilibrium under passive displacements: profiles of axial stress (a), displacements (b) and shear mobilization (c).

4 PILE TUNNEL INTERACTION

The method can also be used with the settlements induced by a tunnel excavation. Consider the same 20 m long, 1 m in diameter, weightless pile, but with a shear resistance that increases linearly with depth ($\gamma`\cdot\beta = 2$ kN/m^3), and a toe capacity of 40% the UBC, which results in a compressive pile capacity of 2.1 MN. The shaft mobilization model considers the following parameters: ($S1 = S3 = 0.050$; S2 = 0.025; $\tau_{ep} = 0.50$). The toe mobilization model is defined with $\Delta\sigma_T = 100$ mm, $\chi = 0.3$ and Rb = 0.1. The predicted load-settlement curve for a full load-unload cycle is presented in Fig. 4a.

The tunnelling settlements can be estimated with the analytical solution of [13], which was derived for a homogeneous undrained clay layer, and assumes that the lining is in contact with the tunnel invert, where there are no ground deformations. This is represented through an equivalent undrained ground loss that models the non-uniform radial convergence of the soil into the oval-shaped soil-lining gap, which sets the displacement field around the tunnel.

The ground displacement at any point (x, z) can be calculated with:

$$\delta(x,z) = VL.R^2.\exp\left[\frac{-1.38x^2}{(Z_t+R)^2} - \frac{0.69z^2}{{Z_t}^2}\right].\left\{\frac{(Z_t-z)}{(z-Z_t)^2+x^2} + \frac{(Z_t-z)(3-4\nu)}{(z+Z_t)^2+x^2} + \frac{[(z+Z_t)^2-x^2].2z}{[(z+Z_t)^2+x^2]^2}\right\} \tag{1}$$

where VL is the volume loss, R is the tunnel radius, x is the horizontal coordinate, Zt is the depth of the tunnel centre, z is the vertical coordinate, and ν is the Poisson's ratio.

Consider a tunnel with a diameter of 10 m, centred at a depth of 30 m. The ground is considered incompressible (ν = 0.5), and the volume loss is set at 1%. The surface settlements and the settlements along the pile, when located right above (Ld = 0), and 10 m to the side of the tunnel alignment (Ld = 10 m), are presented in Fig. 4b.

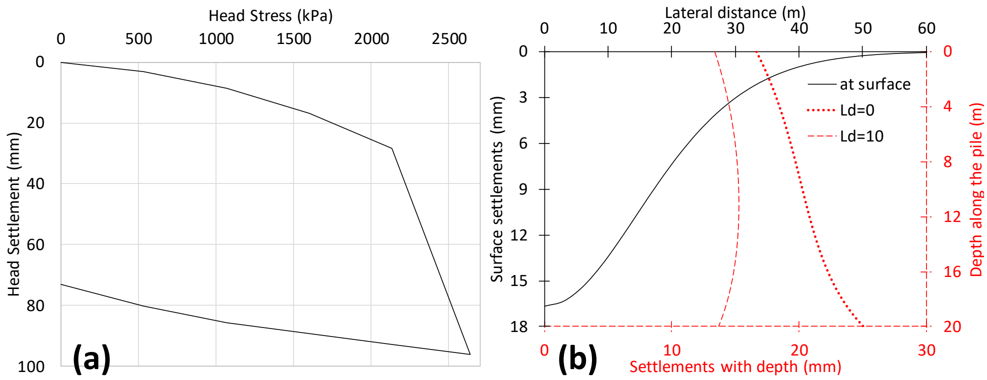

Figure 4: Load settlement curve (a) and tunnelling induced settlements (b).

If these two settlement profiles are imposed to the pile when it is loaded to 50% UBC, the responses are fundamentally different. Right above the tunnel, the tunnelling induced settlements (PD) basically increase with depth, which causes the pile to settle from 12 mm when loaded, to about 36 mm when subjected to the PD. When the pile is to the side of the tunnel, the settlements are more uniform with depth, causing a pile settlement of 26 mm (Fig. 5a). From the initial shear mobilization, the new relations between the ground and the pile settlement, leads to a different state of equilibrium. Above the tunnel, the shear mobilization is increased, particularly close to the surface. To the side of the tunnel, the mobilization decreases around the central half of the pile, and increases around the extremities (Fig. 5b).

These variations in the shear mobilization cause new distributions of axial stress along the pile body. For that, the mobilization levels are combined with the magnitude of the maximum shear resistance at each point. The results basically indicate that a decrease when the pile is on top of the tunnel, and an increase when it is at the side (Fig. 5c,d).

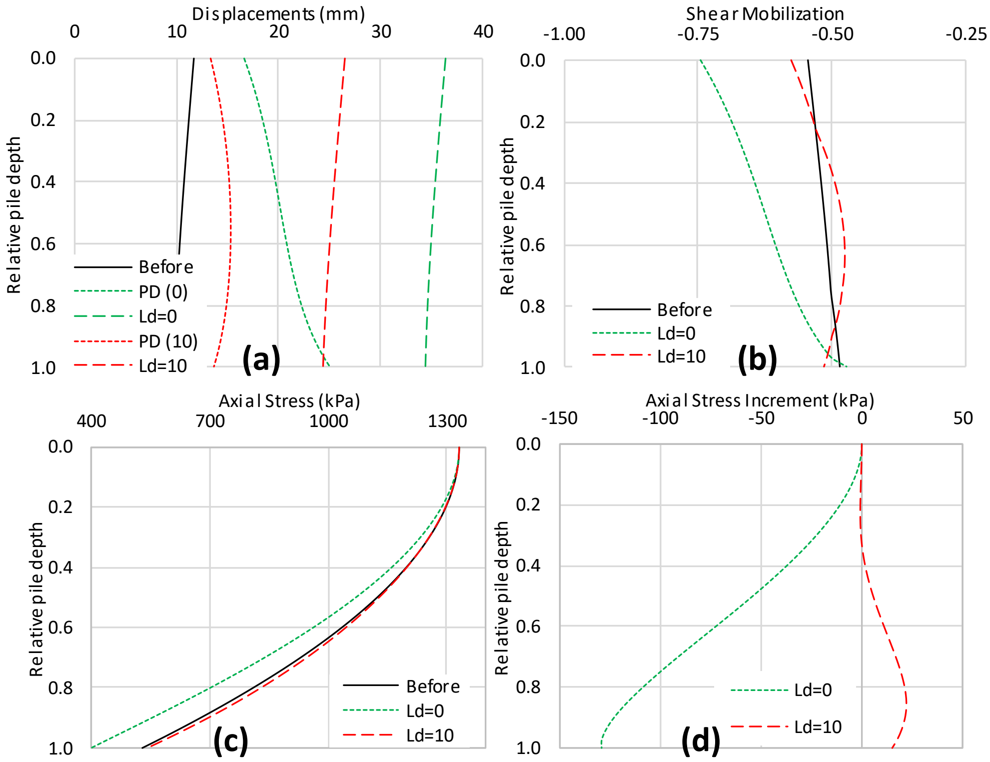

Figure 5: Profiles of displacements (a), shear mobilization (b), axial stress (c) and axial stress increment (d) along the pile.

These results agree qualitatively with what has been described in the literature, namely that the pile settlements are inversely proportional to their lateral distance to the tunnel, and that negative friction is induced when the pile is at a certain distance from the tunnel alignment [1].

5 CONCLUSION

This paper presented how a modified version of the load transfer method can be used to predict how a single pile reacts when subjected to passive displacements. Even though pile groups and the rigidity of the superstructure cannot be directly assessed, the system can predict the new state of equilibrium for an imposed head displacement. This can be used to indirectly represent the load redistribution in a system with multiple bearing elements. Typical profiles of settlements induced by deep excavations and tunnels have been analysed to illustrate the interaction mechanism. The results agree with the literature and offer a rational framework to understand the different pile responses around a tunnel excavation.

REFERENCES

[1] T. G. S. Dias and A. Bezuijen, *Data Analysis of Pile Tunnel Interaction*, J. Geotech. Geoenvironmental Eng. **141** (2015).

[2] T. G. S. Dias, *Pile tunnel interaction*, PhD Thesis, Ghent University (2017).

[3] T. G. S. Dias and A. Bezuijen, *Pile-Tunnel Interaction: A conceptual analysis*, in: Proceedings of the 8th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, Seoul, South Korea (CRC Press, Leiden, 2014).

[4] T. G. S. Dias and A. Bezuijen, *A Load Transfer Method for Piles Under Axial Loading and Unloading*, J. Geotech. Geoenvironmental Eng, submitted.

[5] H. M. Coyle and L. C. Reese, *Load Transfer for Axially Loaded Piles in Clay*, J. Soil Mech. Found. Eng. Div. ASCE **92** (1966) 1–26.

[6] H. G. Poulos and E. H. Davis, Pile foundation analysis and design (1980).

[7] F. Massad, *The analysis of piles considering soil stiffness and residual stresses*, in: Proceedings of the 10th Pan-American Conf. Soil Mech. Found. Eng., Guadalajara, Mexico (1995).

[8] A. Viana da Fonseca, J. A. Santos, E. C. Esteves, and F. Massad, *Analysis of Piles in Residual Soil from Granite Considering Residual Loads*, Soils and Rocks **30** (2007) 63–80.

[9] F. Massad, *Theoretical and Experimental Studies on the Resilience of Driven Piles*, Soils and Rocks **37** (2014) 113–132.

[10] M. Korff, *Response of piled buildings to the construction of deep excavations*, PhD Thesis, University of Cambridge (2012).

[11] A. Franza, S. Idinyang, C. M. Heron, and A. M. Marshall, *Development of a coupled centrifuge-numerical model to study soil-structure interaction problems*, in: Proceedings of the 3rd European Conference on Physical Modelling in Geotechnics, Nantes, France (2016).

[12] A. Franza and A. M. Marshall, *Centrifuge modelling of piled structure response to tunneling*, in: Proceedings of the 3rd European Conference on Physical Modelling in Geotechnics, Nantes, France (2016).

[13] N. Loganathan and H. G. Poulos, *Analytical prediction for tunneling-induced ground movements in clays*, J. Geotech. Geoenvironmental Eng. **124** (1998) 846–856.